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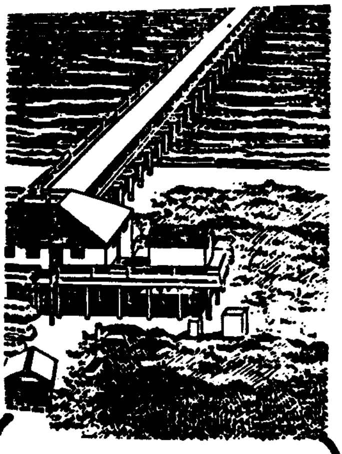

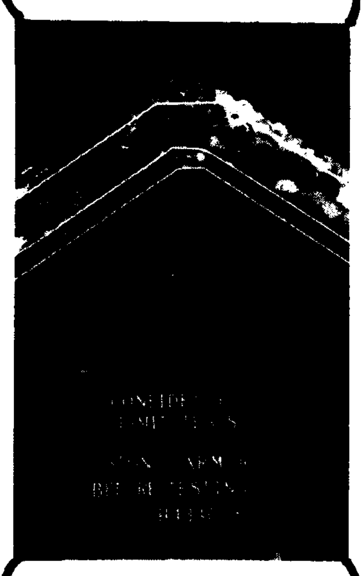

TECHNICAL REPORT CERC-91-17

(2)

INVESTIGATION OF RANDOM VARIATIONS IN STABILITY RESPONSE OF STONE-ARMORED, RUBBLE-MOUND BREAKWATERS

by

Robert D. Carver, Brenda J. Wright

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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13. ABSTRACT *(Maximum 200 words)*

The purpose of this investigation was to obtain a better understanding of variations in the stability response of stone armor when used on breakwater trunks. More specifically, the goal was to quantify the random variations that may occur from one test to another.

Based on results of model tests described herein, it was determined that breakwater stability may be greatly affected by random variations in testing; thus, repeat testing is a must. Also, test results clearly show the influence of wave period with the lower stabilities occurring at the lower values of d/L, i.e., longer wave periods in shallower water.

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Preface

Authority for the US Army Engineer Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC), to conduct this study was granted by Headquarters, US Army Corps of Engineers (HQUSACE), under Work Unit 32534, "Breakwater Stability - A New Design Approach," Coastal Structure Evaluation and Design Program, Coastal Engineering Area of Civil Works Research and Development. The HQUSACE Technical Monitors for this research were Messrs. John H. Lockhart, Jr.; John G. Housley; James E. Crews; and Robert H. Campbell. The CERC Program Manager was Dr. C. Linwood Vincent.

The study was conducted by personnel of CERC under the general direction of Dr. James R. Houston, Chief, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC. Direct supervision was provided by Messrs. C. E. Chatham, Chief, Wave Dynamics Division (WDD), and D. Donald Davidson, Chief, Wave Research Branch (WRB), WDD. This report was prepared by Mr. Robert D. Carver, Principal Investigator, and Ms. Brenda J. Wright, Engineering Technician, WRB. The model was operated by Ms. Wright.

COL Larry B. Fulton, EN, was Commander and Director of WES during report publication. Dr. Robert W. Whalin was Technical Director.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

1 Introduction

Background

During the past decade, much consternation has arisen in the international coastal engineering community over the use of the Hudson Stability Equation (*Shore Protection Manual* 1984). Most researchers have the highest respect for the pioneering work accomplished by Hudson during the 1950's and 1960's; however, based on a detailed study of the original work, conversations with Mr. Hudson, and an attempt to understand the physics of the problem, many researchers have concluded that the present formula does not necessarily address all design parameters. Because the stability coefficient (K_D)[1] combines the effects of over 30 wave and structure variables, it is reasonable to expect that K_D may vary from one investigation to another (as confirmed by recent laboratory tests), especially for shallow-water conditions.

Tests conducted by Carver (1983) using depth-limited monochromatic breaking waves on stone and dolos produced the following conclusions:

a. Armor stability is influenced by wave steepness (H/L), Ursell Number (L^2H/d^3), relative wave height (H/d), and breakwater slope.

b. Effects of H/d, L^2H/d^3, and H/L are more pronounced for dolos armor.

c. In general, minimum stability for each armor type occurred for the larger values of H/d, intermediate values of H/L, and larger values of L^2H/d^3.

d. Linear Hudson-type data fits generally give a reasonable approximation of the stability number as a function of breakwater

1 For convenience, symbols and abbreviations are listed in the Notation (Appendix A).

slope; however, the influences of H/d, H/L, and L^2H/d^3 are strong enough to merit their consideration in selection of armor unit weight.

Based on these conclusions, Carver (1983) recommended that armor stability for breaking waves be presented as a function of wave height, wave period, and water depth (e.g., Ursell Number).

Carver and Wright (in preparation) reanalyzed 26 site-specific model studies in which tetrapod, tribar, dolos, and stone armor were used on breakwater trunks and heads. They found stability to be dependent on the combined effects of wave height, wave period, and water depth with minimum stability occurring at the lower values of relative depth (d/L) and higher values of H/d, i.e., longer wave periods in shallower water. Their findings for rough angular stone armor with breakwater slope ranging from 1:1.5 to 1:2.5 are shown in Figure 1.

Purpose of Study

The purpose of the present investigation is to obtain a better understanding of variations in the stability response of stone armor when used on breakwater trunks. More specifically, the goal is to quantify the random variations that may occur from one test to another and thus augment the data presented in Figure 1.

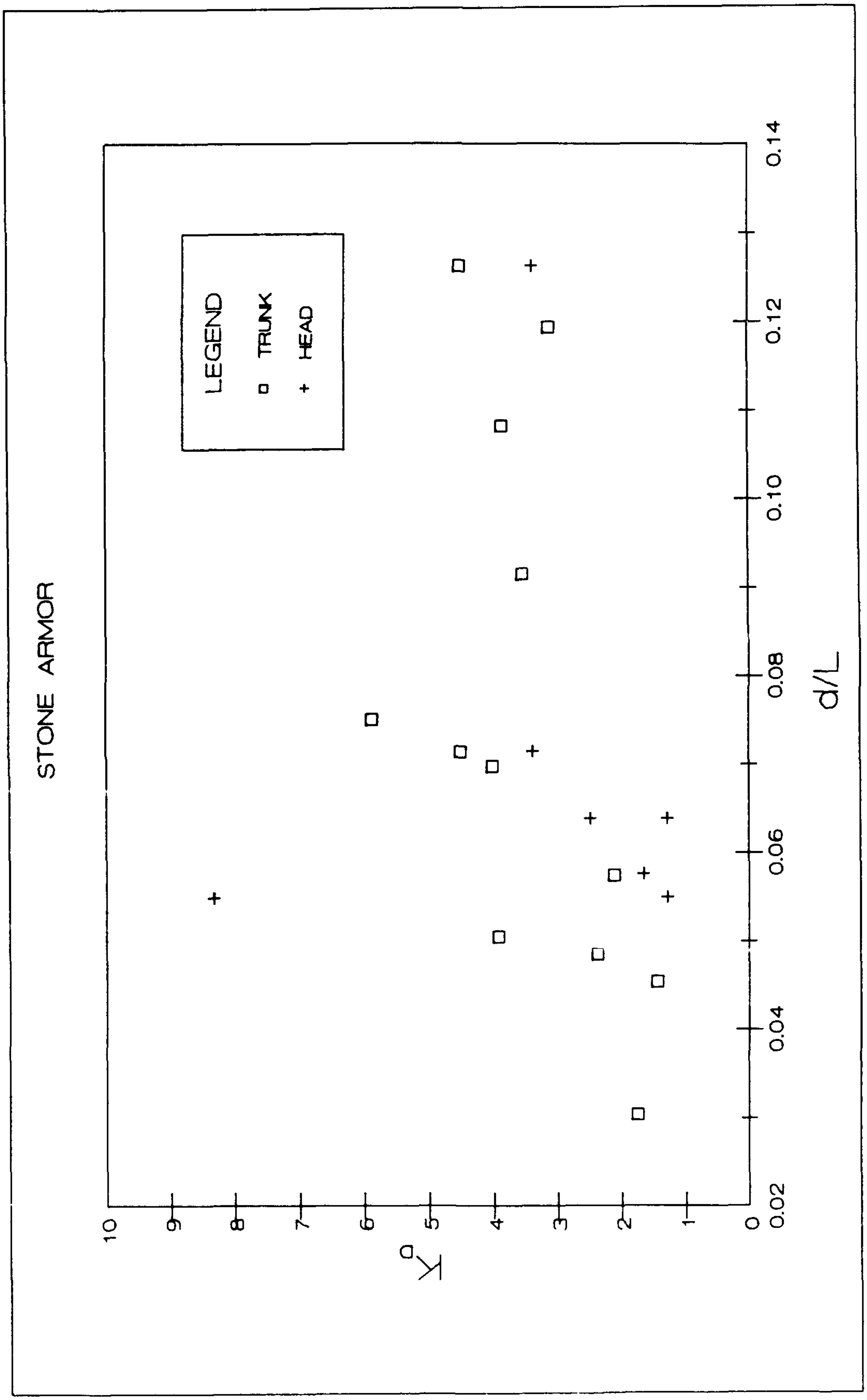

Figure 1. Stability coefficient versus *d/L* for previous site-specific studies

2 Tests

Stability Scale Effects

If the absolute sizes of experimental breakwater materials and wave dimensions become too small, flow around the armor units enters the laminar regime, and the induced drag forces become a direct function of the Reynolds number. Under these circumstances, prototype phenomena are not properly simulated, and stability scale effects are induced. Hudson (1975) presents a detailed discussion of the design requirements necessary to ensure the preclusion of stability scale effects in small-scale breakwater tests and concludes that scale effects will be negligible if the Reynolds stability number (R_N)

$$R_n = \frac{g^{1/2} H^{1/2} l_a}{\nu} \qquad (1)$$

where

g = acceleration due to gravity, ft/sec^2

H = wave height, ft

l_a = characteristic length of armor unit, ft

ν = kinematic viscosity

is equal to or greater than 3×10^4. For all tests reported herein, the sizes of experimental armor and wave dimensions were selected such that scale effects were insignificant (i.e., R_N was greater than 3×10^4).

Method of Constructing Test Sections

All experimental breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing full-scale breakwaters. The core material was dampened as it was dumped by bucket or shovel into the flume and was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype structure. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. The underlayer stone then was added by shovel and smoothed to grade by hand or with trowels. Armor units used in the cover layers were placed in a random manner corresponding to work performed by a general coastal contractor; i.e., they were individually placed but were laid down without special orientation or fitting. After each test the armor units were removed from the breakwater, all of the underlayer stones were replaced to the grade of the original test section, and the armor was replaced. Armor units and the first underlayer material were placed in two layers, and the number of armor units per given area was equal to that presently recommended for new construction in EM 1110-2-2904 (1986).

Test Equipment and Materials

Equipment used

Tests were conducted in a concrete wave flume, 11 ft[1] wide, 6 ft deep, and 245 ft long. The cross section of the tank in the vicinity of the structures was partitioned into two 3-ft-wide channels and two 2.5-ft-wide channels (Figure 2). Identical test sections were constructed in the 3-ft channels while wave absorption was achieved in the 2.5-ft channels, which were left empty. The flume is equipped with an electro-hydraulic, horizontal-displacement wave generator capable of producing monochromatic and irregular waves of various periods and heights. Changes in water surface elevation as a function of time (wave heights) were measured by electrical capacitance-type gages at selected locations. The wave machine was controlled by and data were collected with an on-line Dec MicroVax I computer. Data were then transferred to a Vax 3600 for analyses.

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page vii.

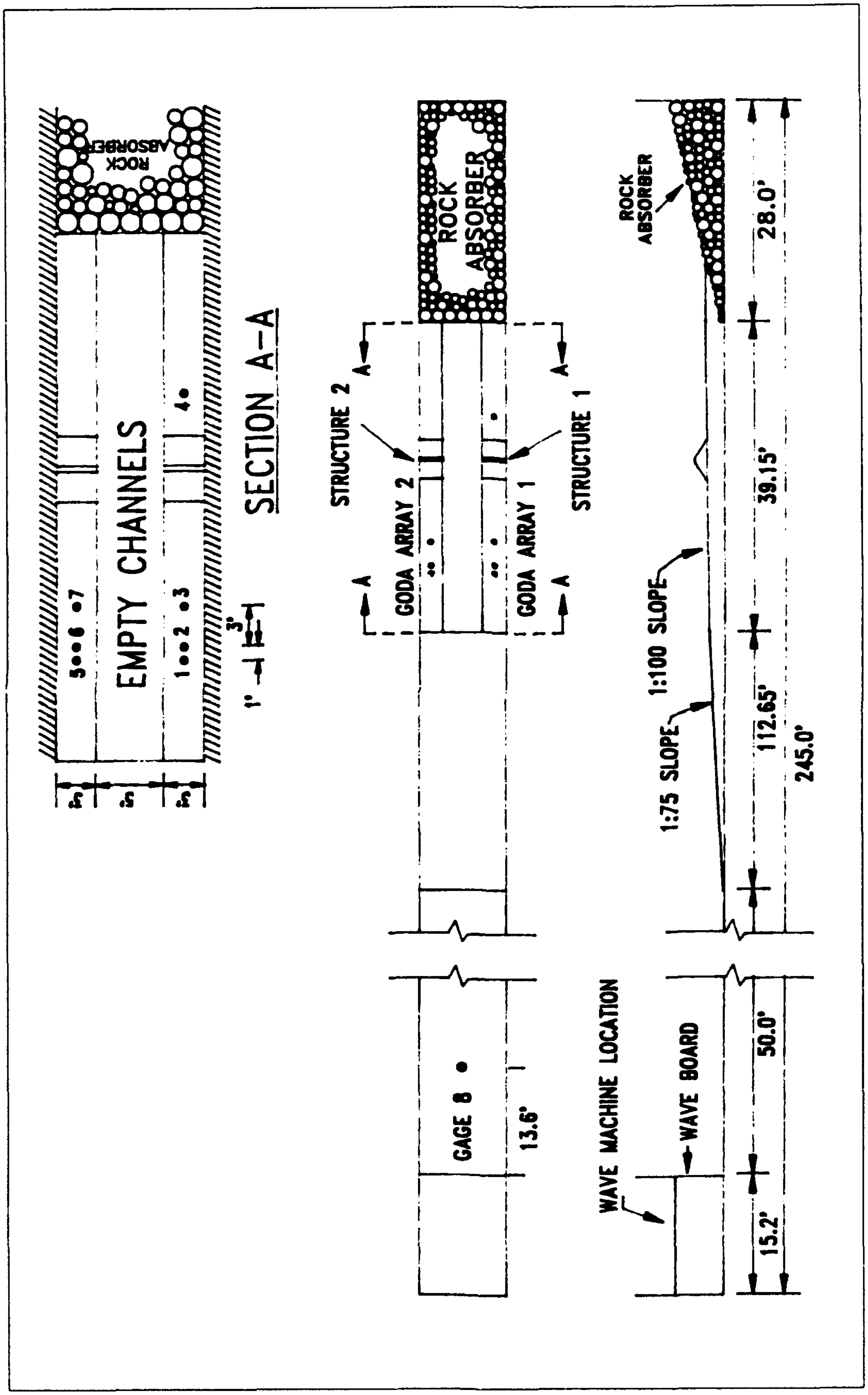

Figure 2. Wave tank cross section

Materials used

Rough hand-shaped granitic stone (W_a) with an average length of about two times its width, average weight of 0.38 lb, and a specific weight of 167 pcf was used to armor the stone sections. Sieve-sized limestone (unit weight = 165 pcf) was used for the underlayers and core.

Selection of Test Conditions

All tests were conducted with a Texel, Marsen, Arsloe (TMA) spectrum. The wave flume was calibrated for periods of 1.5, 2.25, 3.0, and 4.0 sec, thus assuring that a wide range of relative depths (d/L's) would be available for testing. Wave period water depth combinations were chosen consistent with the range of d/L values encountered in the site-specific investigations summarized by Carver and Wright (in preparation). It should be noted that the majority of tests upon which present general design guidance is based were conducted in a d/L range outside the limits of conditions to which prototype structures are typically exposed. Goda and Suzuki's (1976) method was used to resolve the incident and reflected spectra.

All tests were conducted on stone sections of the type shown in Figure 3 and Photos 1 and 2. Both the sea- and beach-side slopes were held constant at 1V:1.5H. Water depth at the toe of the structures was 0.80 ft for all tests.

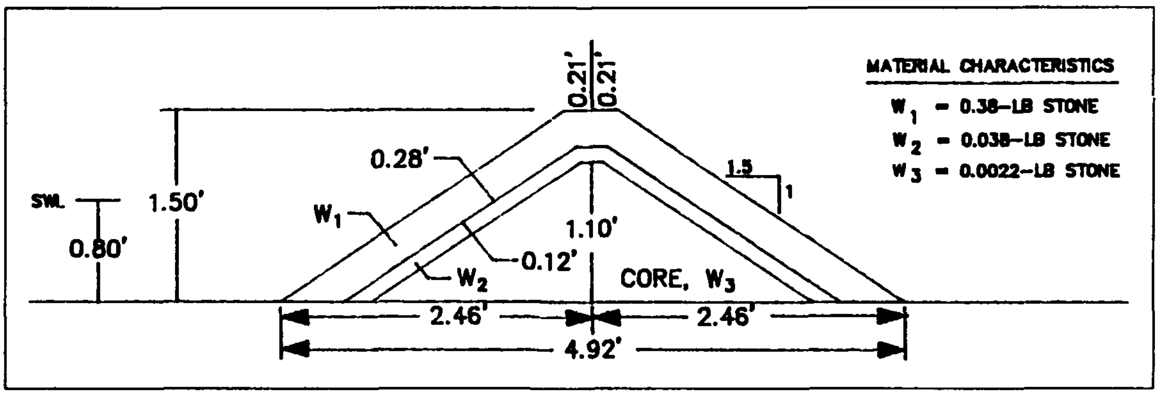

Figure 3. Typical breakwater cross section

Design wave heights for the no-damage criterion were determined by subjecting the test sections to irregular waves successively larger in height in 0.01- to 0.02-ft increments until the maximum heights for which the armor was stable were reached. Each was allowed to attack the breakwater for a time equivalent to at least 1,000 peak wave periods; then the test sections were rebuilt prior to attack by the next added increment wave. This 1,000 wave duration allowed sufficient time for a statistically stable irregular wave condition to develop in the wave tank and also was sufficient for the test sections to stabilize. Acceptability of the final condition (damage accessment) of each test section was based on observations by experienced engineers and technicians learned in the damage/no-damage criteria.

3 Test Results

General

Stability test results are summarized in Tables 1 and 2. Presented therein are test conditions of peak wave period T_p, water depth d at the toe of the structure along with experimentally determined design wave heights H_{mo}, and corresponding stability coefficients (K_D) and relative depth (d/L). Six or seven repeat tests were conducted for each wave period investigated. Photos 3-8 show typical after-testing views of the structures. As evidenced in these photos, the design wave conditions allowed occasional displacement of a few random armor units; however, movement was never extensive enough to jeopardize the stability of the test sections.

The stability number N_s and stability coefficient K_D provide a way to correlate stability test results. The following definition is used for stability number and stability coefficient as applied to tests with irregular waves in this report.

$$N_s = \frac{\gamma_a^{1/3} H_{mo}}{\left(S_a - 1\right) W_a^{1/3}} \tag{2}$$

where

γ_a = specific weight of an armor unit in pcf

H_{mo} = wave height at the structure toe in feet

S_a = specific gravity of an armor unit relative to the water in which it is placed

W_a = weight in pounds of an acceptably stable armor unit

A more detailed discussion of the variable affecting N_s can be found in Carver (1983). The stability coefficient K_D as defined by Hudson (1958) is

$$K_D = \frac{N_s^3}{\cot\alpha} \tag{3}$$

where $\cot\alpha$ is the slope of the structure.

Figures 4 and 5 present K_D and N_s as a function of wave period for Structures 1 and 2, respectively. All results are combined in Figure 6. These data show stability to be influenced by wave period with the lower stabilities being observed at the longer wave periods. Also, the data spread within a wave period is greater than was anticipated at the onset of testing, leading to the conclusion that random variability may have a greater influence on stability than was previously thought.

Previous breakwater stability work has shown relative depth (d/L) to be an important dimensionless variable associated with changes in stability response. Therefore K_D is plotted as a function of d/L in Figure 7, and a strong correlation is observed.

Development of Confidence Limits

By definition, random placement of the armor implies that each building of the structure represents only one outcome of a very large number of possibilities. Thus, the experimentally determined design wave heights and corresponding stability coefficients can be expected to assume a range of values if repeat tests are conducted. As evidenced by the data presented herein, this random variation of stability within a wave period appears to be present. Also, stability appears to systematically decrease with increasing wave period.

If it is assumed that test results are normally distributed within a wave period and there are no significant differences in results obtained from the two structures, standard statistical techniques (Ostle and Mensing 1975) can be applied to determine means, standard deviations, and confidence limits. Statistical analysis of data gathered in this study yielded the following results relative to K_D:

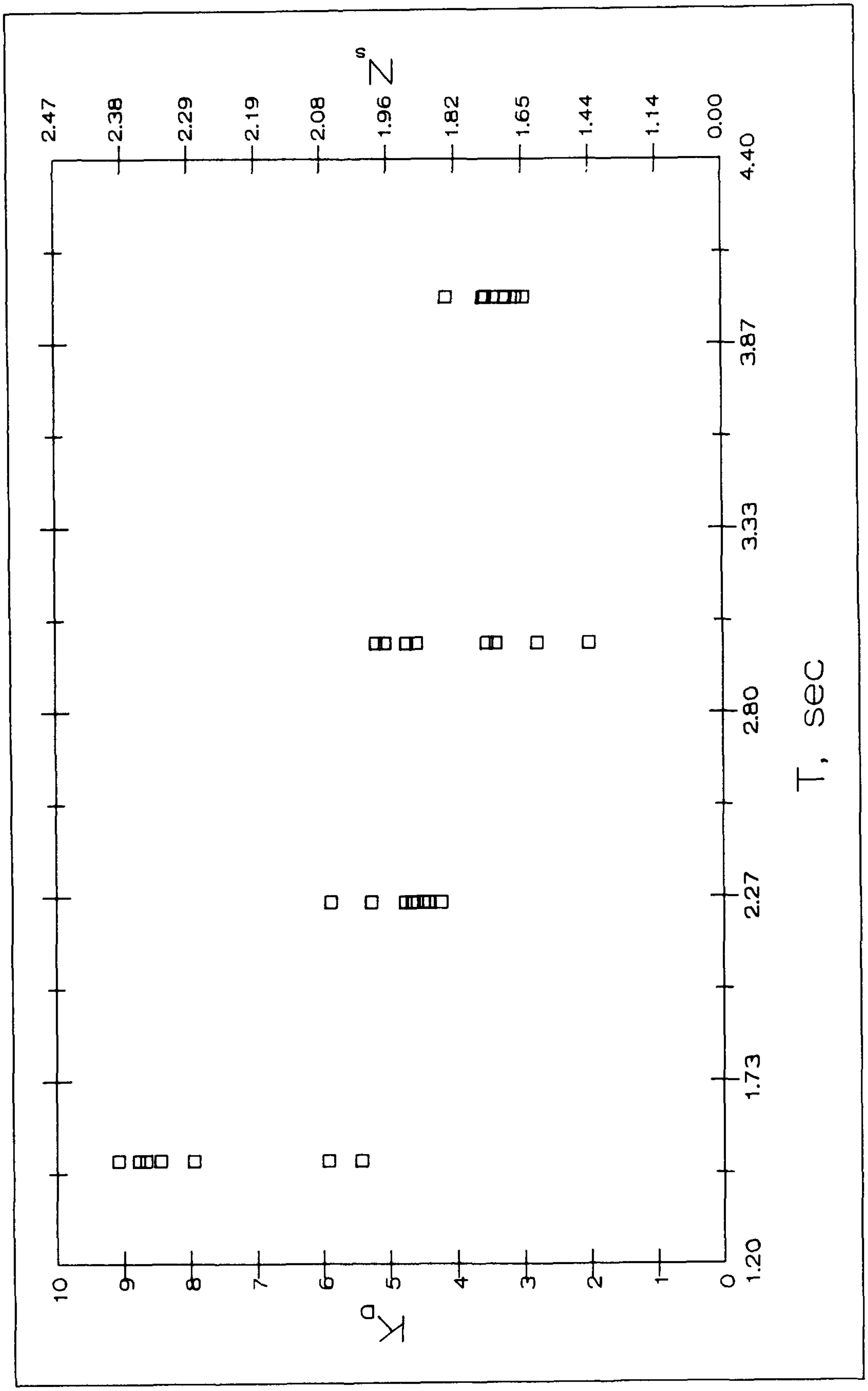

Figure 4. Stability coefficient and stability number versus wave period, Structure 1

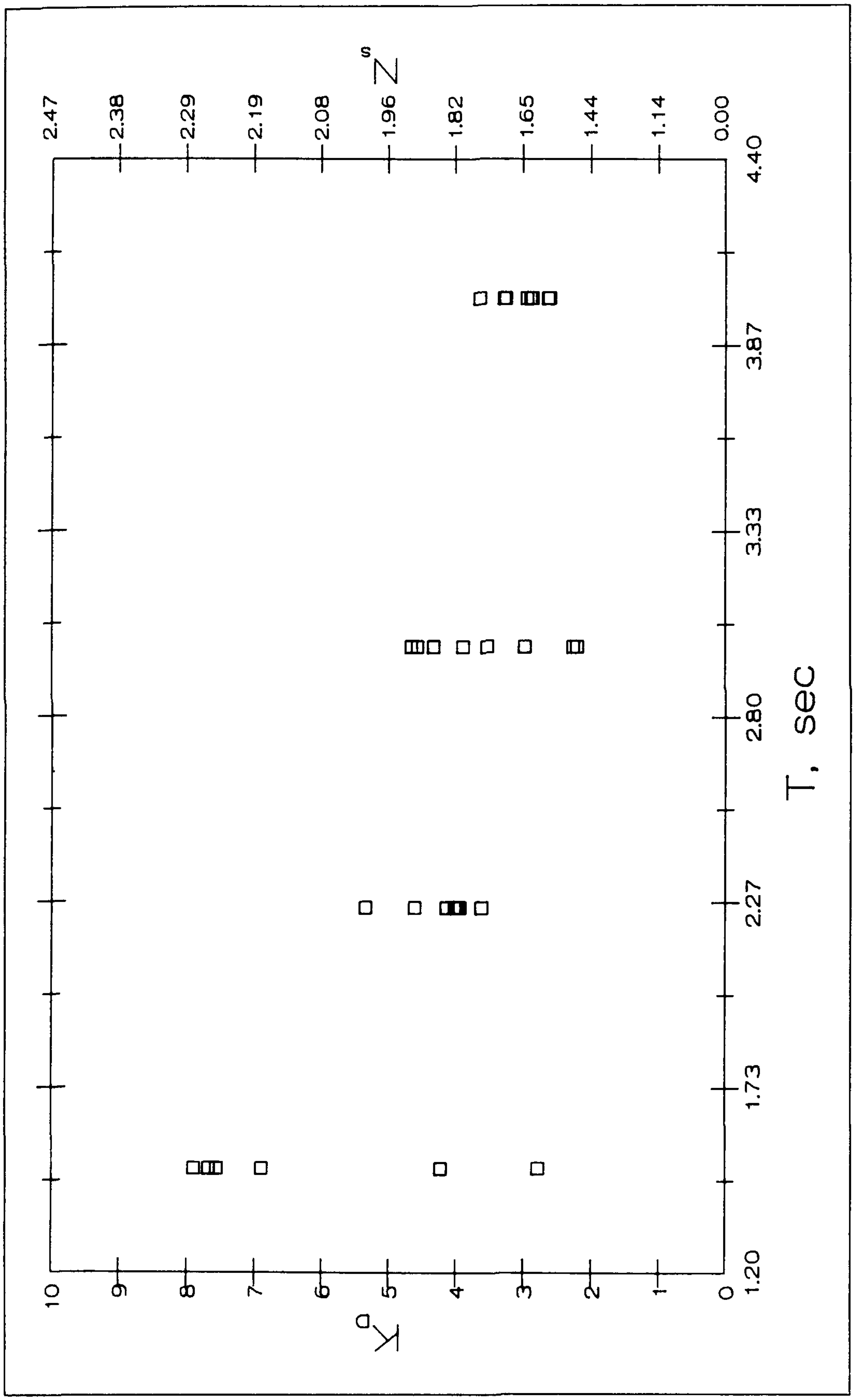

Figure 5. Stability coefficient and stability number versus wave period, Structure 2

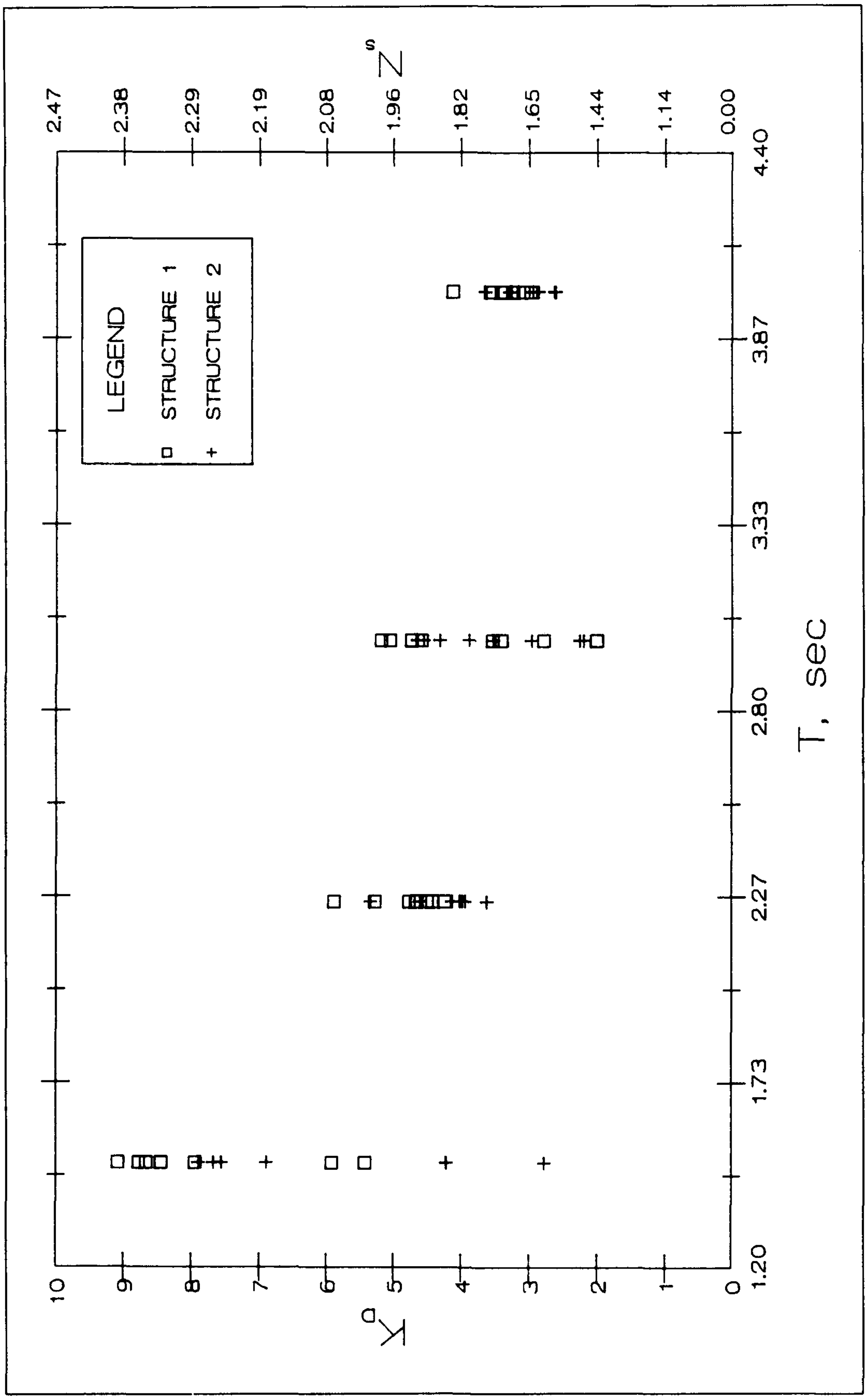

Figure 6. Stability coefficient and stability number versus wave period for Structures 1 and 2

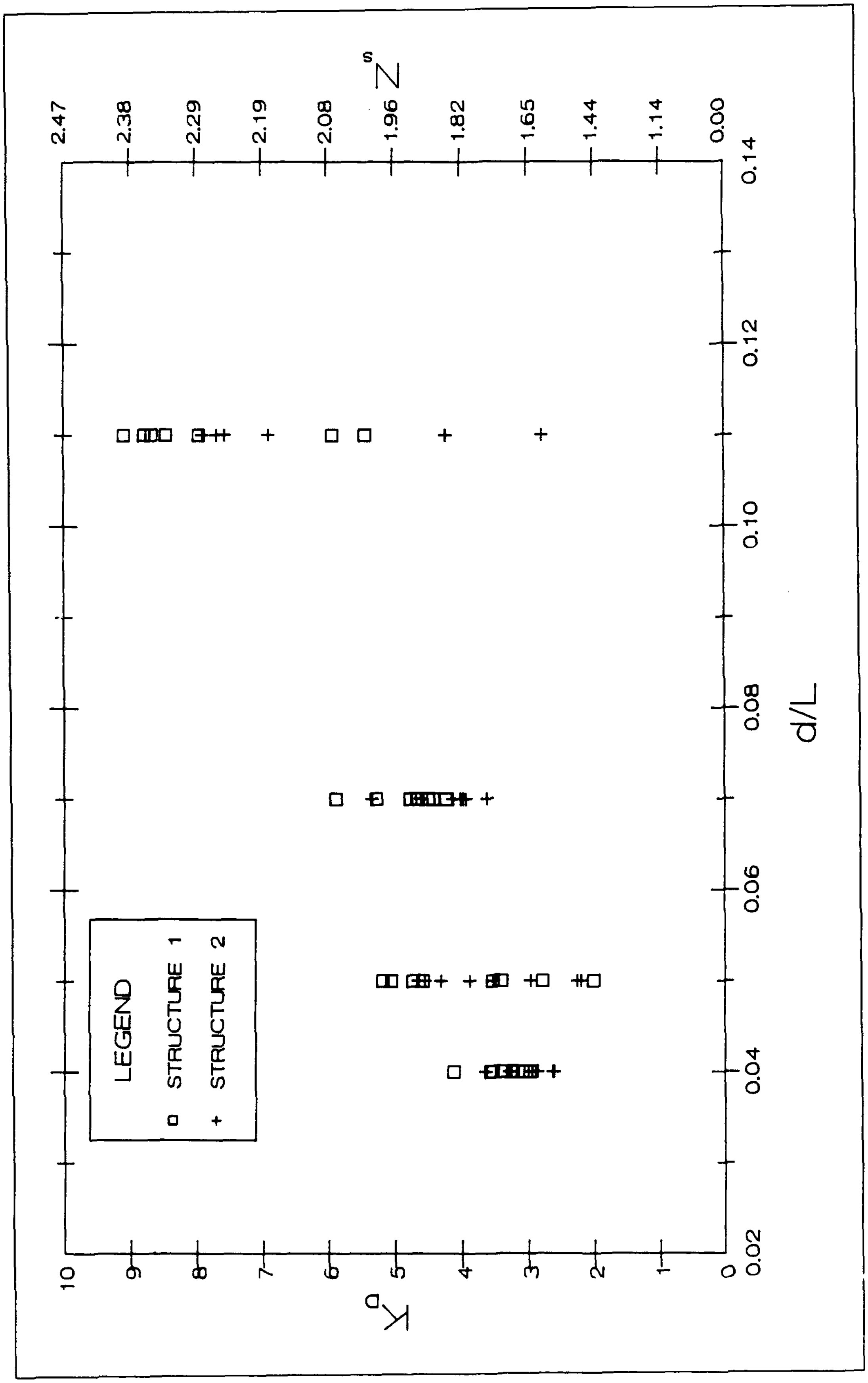

Figure 7. Stability coefficient and stability number versus *d/L* for Structures 1 and 2

T_p, sec	d/L	n	Average K_D	S	L_{90}
1.50	0.11	14	6.8	1.99	5.9
2.25	0.07	16	4.5	0.60	4.2
3.00	0.05	16	3.7	1.05	3.2
4.00	0.04	16	3.2	0.40	3.0

where n, s, and L_{90} are the sample size, standard deviation of K_D, and lower 90-percent confidence limit of K_D, respectively.

The lower 90-percent confidence limit appears to be a reasonable choice for design with the exception of the 1.5-sec wave period. Test results for this period show significantly more scatter than the other periods investigated, and the normal distribution assumption is less valid. Therefore, it was decided to use the average K_D less one standard deviation (6.8 - 1.99 = 4.8) instead of the predicted lower limit value of 5.9. Figure 8 presents lower limit K_D's as functions of d/L.

Discussion

Test results presented herein are very significant in that

a. Breakwater stability may be greatly affected by random variations in testing; thus, repeat testing is a must.

b. They clearly show the influence of wave period with the lower stabilities occurring at the lower values of d/L, i.e., longer wave periods in shallower water.

Earlier tests conducted by Hudson (1958) and Jackson (1968) did not show a strong wave period dependency. This difference probably results from the fact that most of the tests conducted by Hudson and Jackson were in a d/L range of 0.15 to 0.50 where the waves are more linear and the effects of period would not be expected to be as significant.

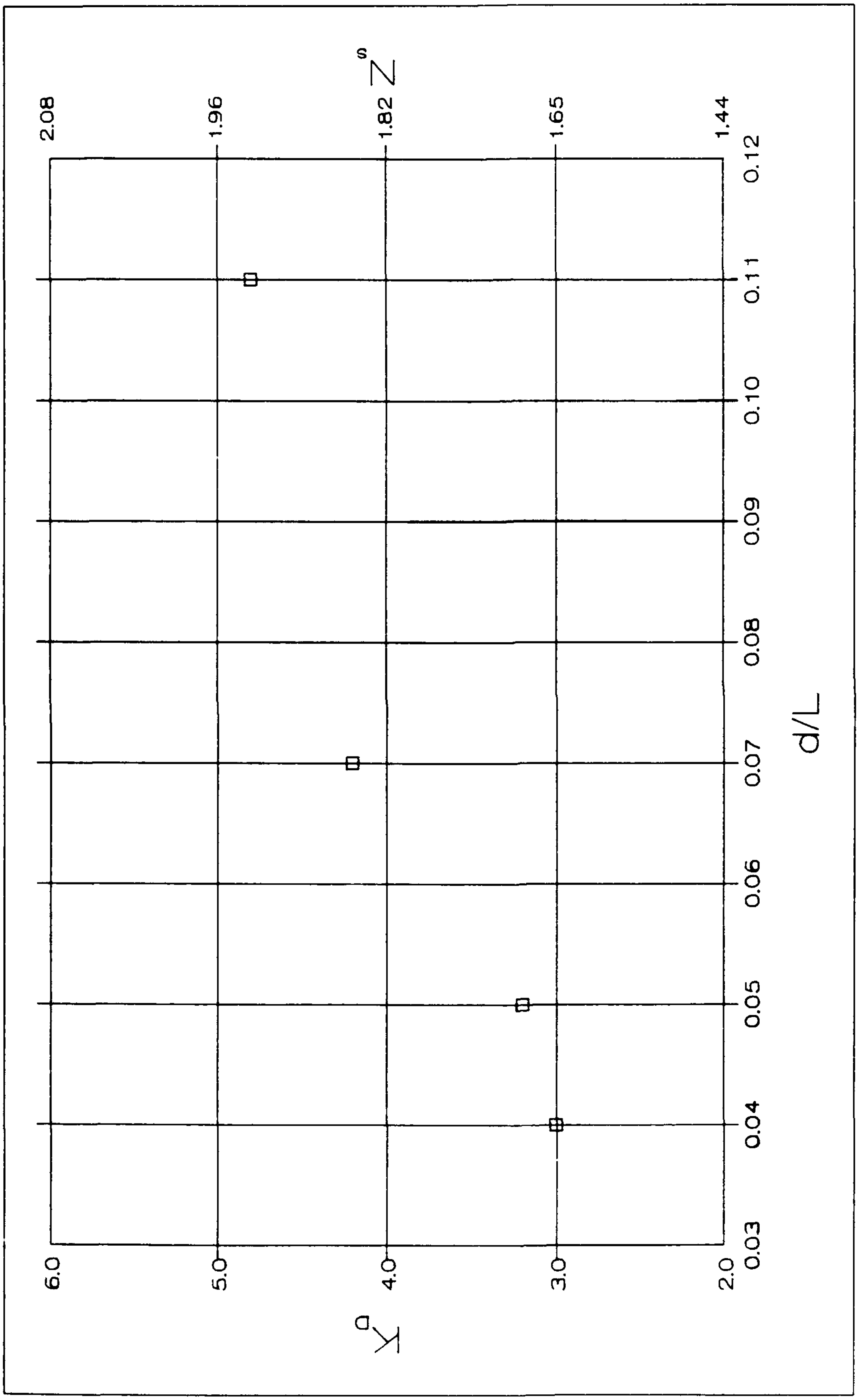

Figure 8. Lower limit stability coefficient and stability number versus *d/L*

4 Recommendation

It is recommended that the design curve presented in Figure 8 be used for the preliminary sizing of armor placed on a 1V:1.5H slope stone since it represents a significant improvement over the single stability coefficient procedure presently used. Also, it is based on results of tests conducted with shallow-water spectra in a *d/L* range typical of actual prototype conditions.

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Table 1
Summary of Stability Test Results, Structure 1

T_p, sec	d/L	H_{mo}, ft	N_s	K_D
1.50	0.11	0.437	2.01	5.42
1.50	0.11	0.450	2.07	5.92
1.50	0.11	0.496	2.28	7.94
1.50	0.11	0.506	2.33	8.44
1.50	0.11	0.510	2.35	8.66
1.50	0.11	0.513	2.36	8.77
1.50	0.11	0.519	2.39	9.08
2.25	0.07	0.402	1.85	4.23
2.25	0.07	0.408	1.88	4.41
2.25	0.07	0.410	1.89	4.49
2.25	0.07	0.415	1.91	4.66
2.25	0.07	0.415	1.91	4.66
2.25	0.07	0.418	1.93	4.76
2.25	0.07	0.433	1.99	5.27
2.25	0.07	0.449	2.07	5.88
3.00	0.05	0.313	1.44	2.01
3.00	0.05	0.350	1.61	2.79
3.00	0.05	0.374	1.72	3.40
3.00	0.05	0.379	1.74	3.54
3.00	0.05	0.413	1.90	4.58
3.00	0.05	0.417	1.92	4.73
3.00	0.05	0.426	1.96	5.05
3.00	0.05	0.430	1.98	5.18
4.00	0.04	0.357	1.64	2.97
4.00	0.04	0.362	1.67	3.09
4.00	0.04	0.368	1.69	3.24
4.00	0.04	0.368	1.69	3.24
4.00	0.04	0.374	1.72	3.40
4.00	0.04	0.379	1.74	3.54
4.00	0.04	0.380	1.75	3.57
4.00	0.04	0.398	1.83	4.12

Note: W_a = 0.38 lb; γ_a = 167 pcf; cotα = 1.5; d = 0.80 ft.

Table 2
Summary of Stability Test Results, Structure 2

T_p, sec	d/L	H_{mo}, ft	N_s	K_D
1.50	0.11	0.349	1.61	2.78
1.50	0.11	0.402	1.85	4.22
1.50	0.11	0.402	1.85	4.22
1.50	0.11	0.473	2.18	6.90
1.50	0.11	0.488	2.25	7.56
1.50	0.11	0.490	2.26	7.68
1.50	0.11	0.495	2.28	7.89
2.25	0.07	0.381	1.76	3.61
2.25	0.07	0.392	1.81	3.93
2.25	0.07	0.394	1.81	3.97
2.25	0.07	0.395	1.82	4.01
2.25	0.07	0.395	1.82	4.01
2.25	0.07	0.399	1.84	4.13
2.25	0.07	0.413	1.90	4.60
2.25	0.07	0.434	2.00	5.34
3.00	0.05	0.323	1.49	2.20
3.00	0.05	0.326	1.50	2.26
3.00	0.05	0.357	1.65	2.97
3.00	0.05	0.378	1.74	3.53
3.00	0.05	0.391	1.80	3.88
3.00	0.05	0.405	1.86	4.31
3.00	0.05	0.412	1.90	4.55
3.00	0.05	0.415	1.91	4.65
4.00	0.04	0.342	1.58	2.61
4.00	0.04	0.343	1.58	2.63
4.00	0.04	0.353	1.63	2.87
4.00	0.04	0.354	1.63	2.89
4.00	0.04	0.356	1.64	2.94
4.00	0.04	0.369	1.70	3.26
4.00	0.04	0.369	1.70	3.28
4.00	0.04	0.382	1.76	3.64

Note: W_a = 0.38 lb; γ_a = 167 pcf; cotα = 1.5; d = 0.80 ft.

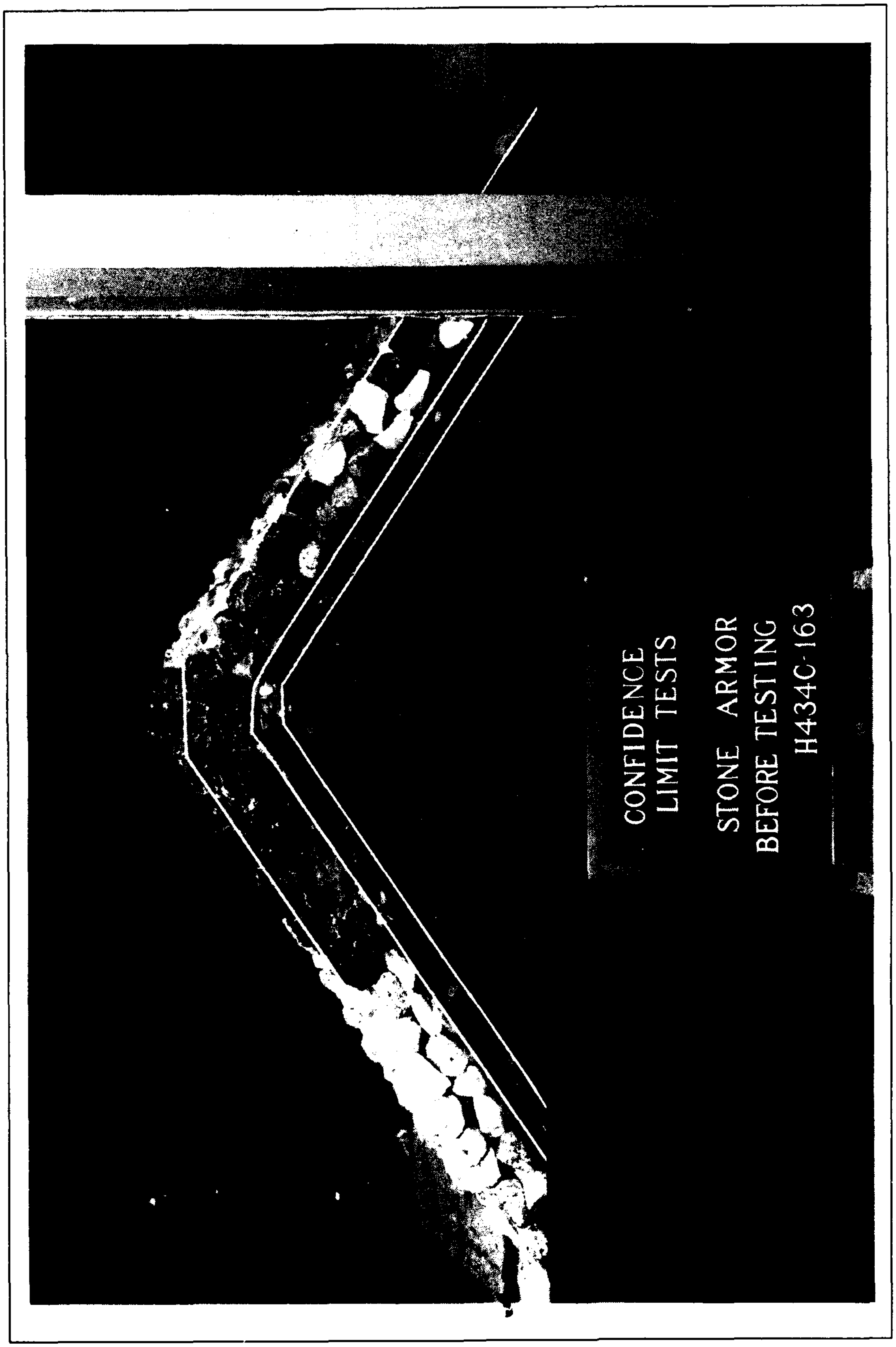

Photo 1. End view of a typical test section before wave attack

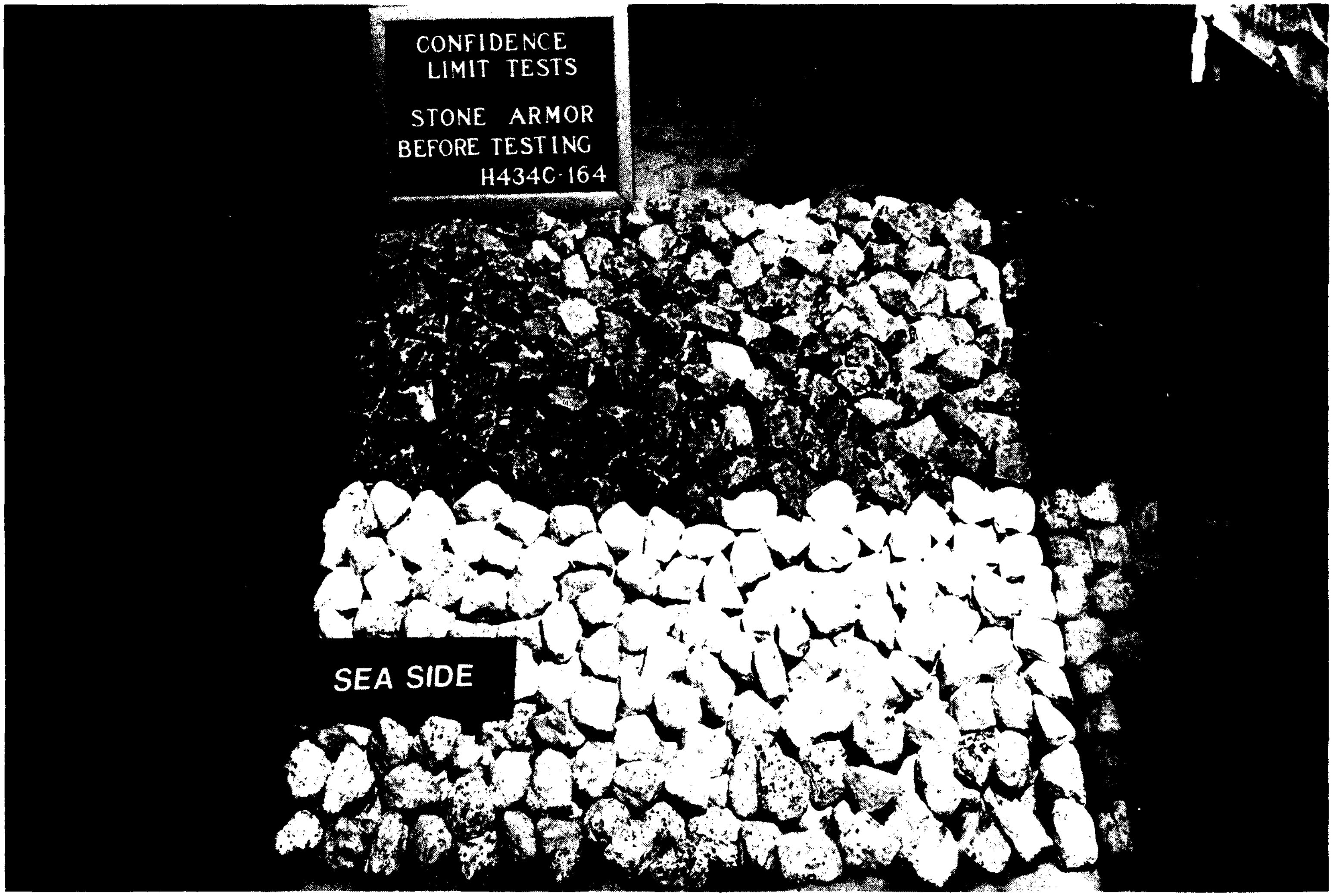

Photo 2. Seaside view of a typical test section before wave attack

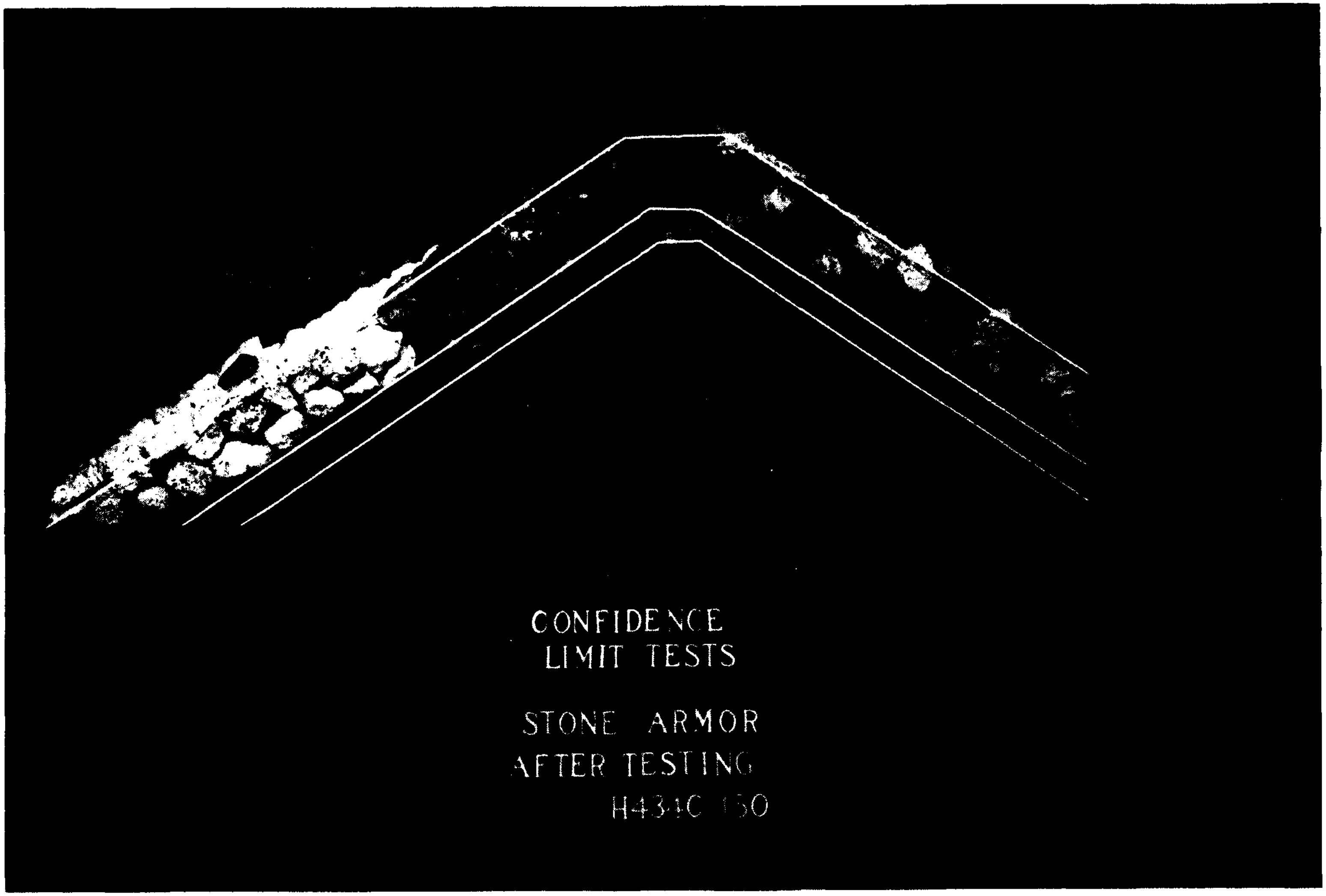

Photo 3. End view of Structure 1 after wave attack

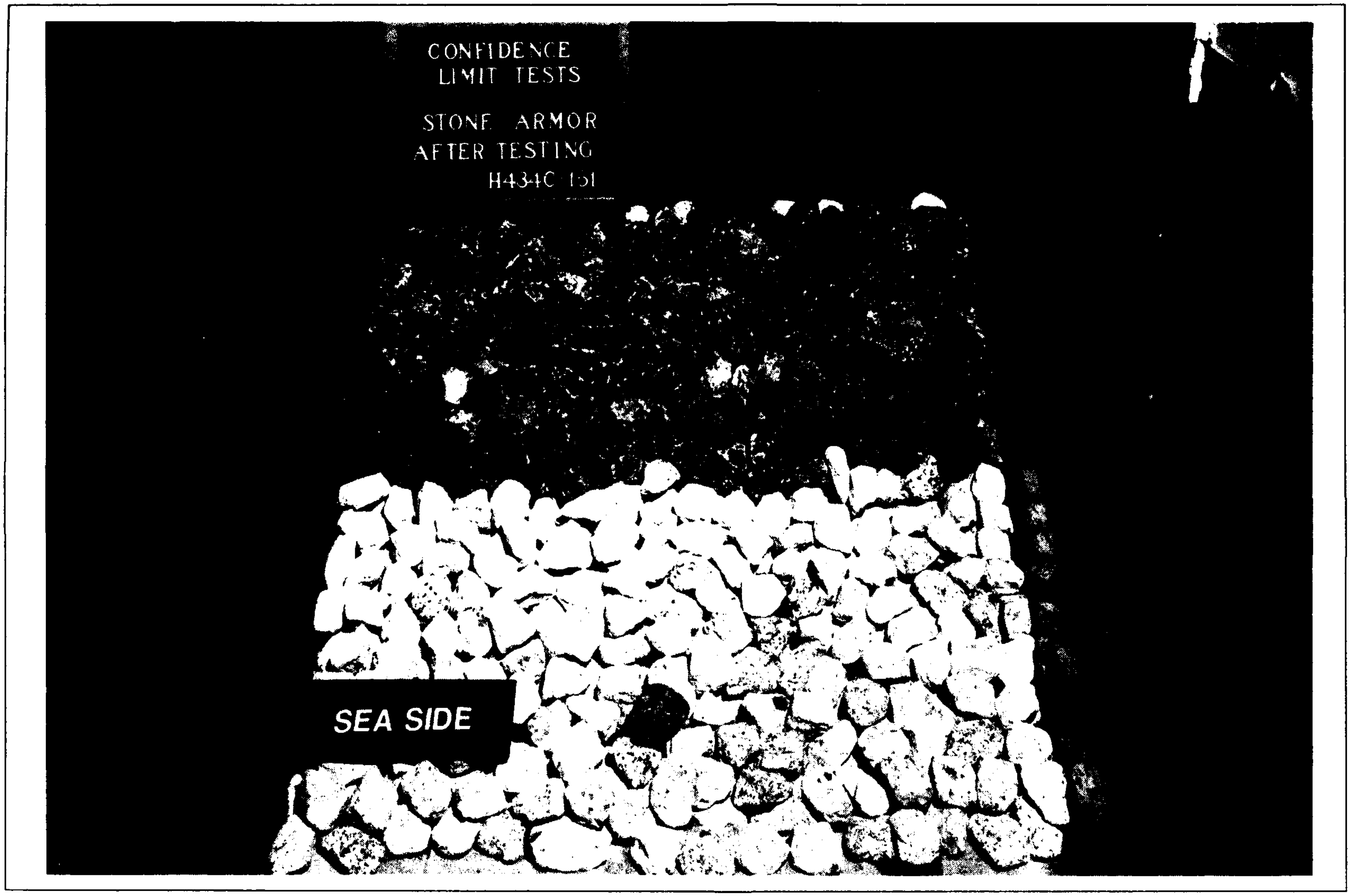

Photo 4. Seaside view of Structure 1 after wave attack

Photo 5. Seaside view of Structure 2 after wave attack

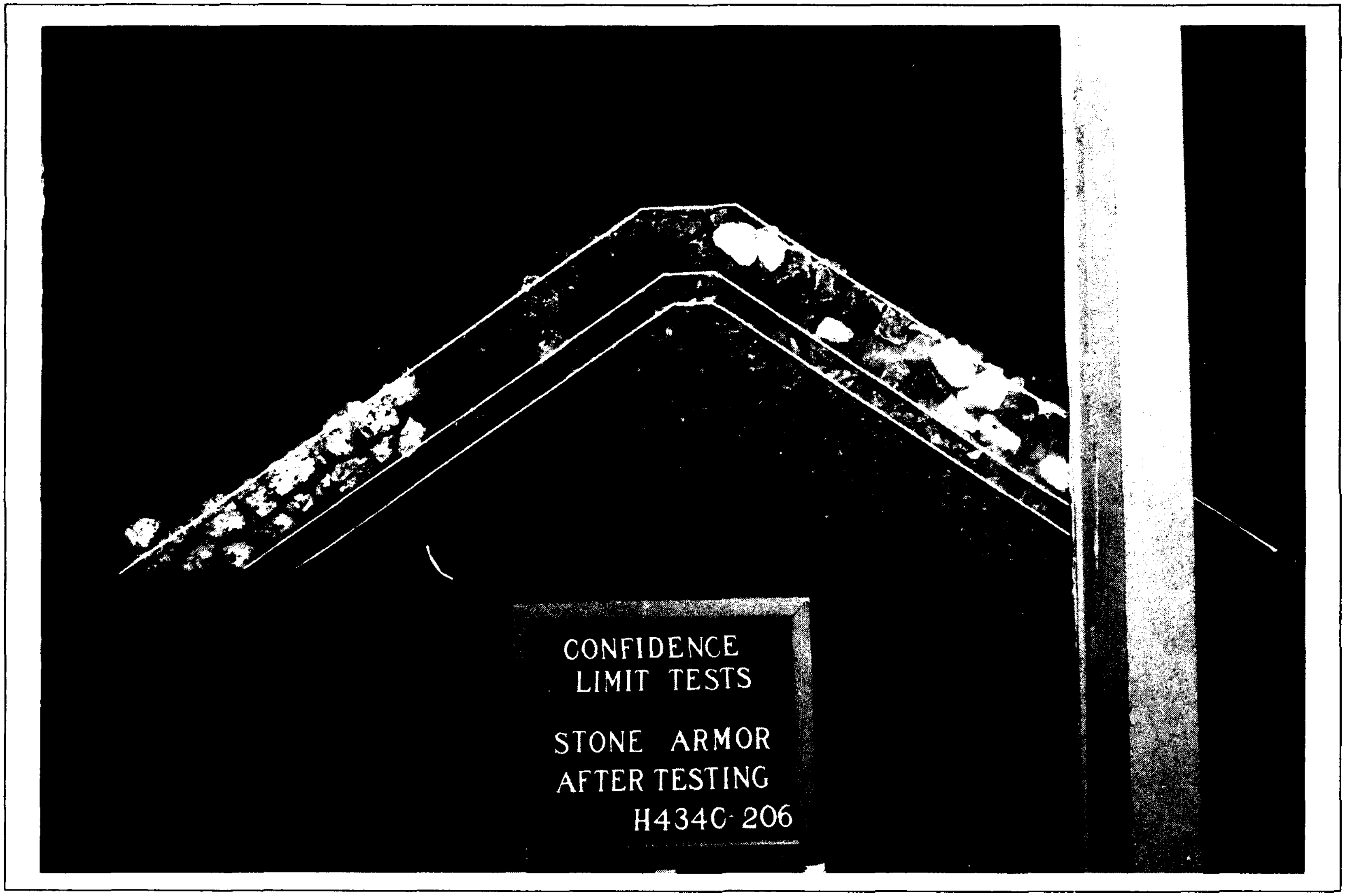

Photo 6. End view of Structure 1 after completion of a typical repeat test

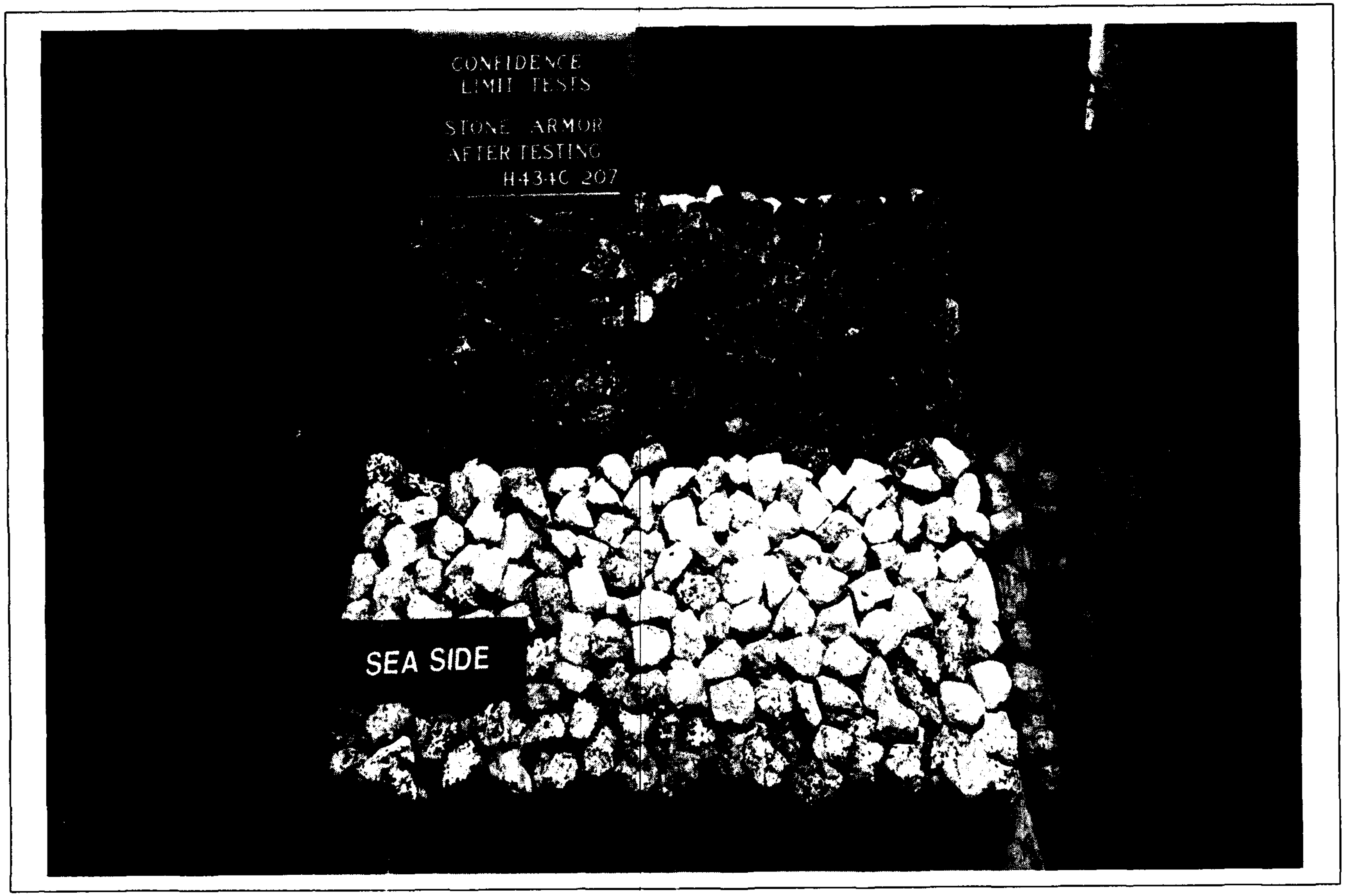

Photo 7. Seaside view of Structure 1 after completion of a typical repeat test

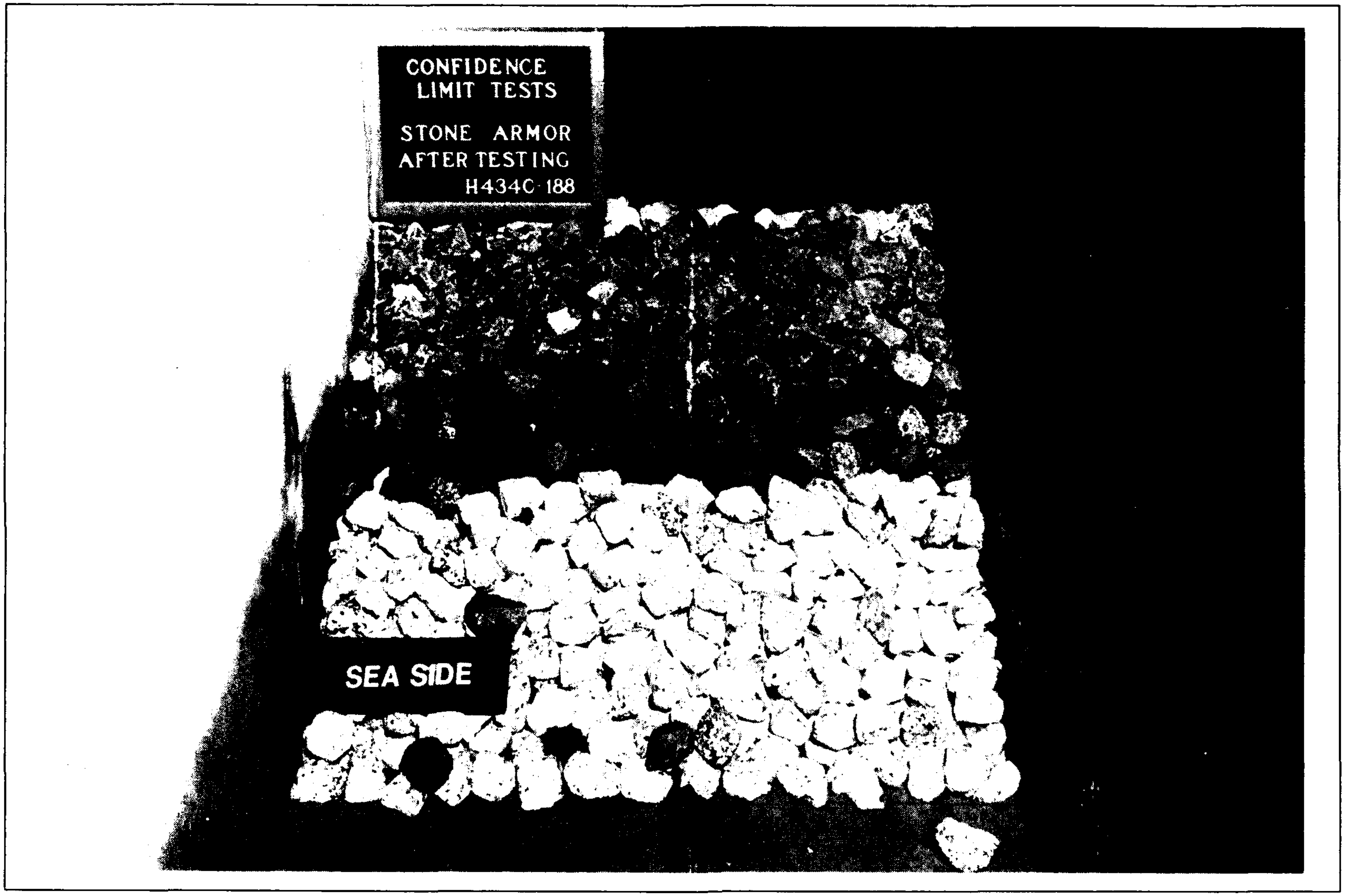

Photo 8. Seaside view of Structure 2 after completion of a typical repeat test

Appendix A
Notation

d	Water depth, ft
d/L	Relative depth, dimensionless
g	Acceleration due to gravity, ft/sec^2
H	Significant wave height, ft, of monochromatic wave train
H_{mo}	Zero-moment wave height, ft, of wave spectrum
H/d	Relative wave height, dimensionless
H/L	Wave steepness, dimensionless
K_D	Hudson stability coefficient, dimensionless
l_a	Characteristic length of armor unit, ft
L^2H/d^3	Ursell number
L_{90}	Lower 90-percent confidence limit
N_s	Stability number
n	Number of tests
R_N	Reynolds stability number
s	Standard deviation of K_D
T_p	Wave period of peak energy density of spectrum, sec
ν	Kinematic viscosity of experimental fluid medium, ft^2/sec
W_a	Weight of individual armor unit, lb

γ_a	Specific weight of armor unit, pcf
α	Angle of structure slope measured from horizontal in degrees
$\cot\alpha$	Slope of structure